Nonlinear Redundancy Analysis of Truss Bridges

Analysis Report

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Acknowledgments

NON-LINEAR ANALYSIS OF TRUSS BRIDGE

This report describes the results of the redundancy analysis of a typical simply supported truss bridge superstructure and its ability to continue to carry vehicular loads beyond the elastic limit. The object of the analysis is to investigate the residual capacity of the structure above the design load and the ensuing nonlinear behavior.

The analysis is performed with the commercial software ABAQUS. The 3D model is shown in Figure 1. Each member of the bridge is modeled with 2D shell elements. The materials used in this analysis are grade 36 ksi steel for the truss members and the deck beams. The concrete deck is modeled using a strength f'$_c$=4.0 ksi. The structural steel is assumed to be nonlinear, while the concrete is assumed to remain elastic as the deck's contribution to the strength of the structural system is not taken into consideration.

The commercial software ABAQUS is used for the structural analysis because of its flexibility and its capability to account for material nonlinearity using advanced models. The analysis is performed for the load location that will give the most critical system response. For all the cases considered, that position coincided with the truck being placed near the middle of the span. This also assured that the same baseline is used in the comparison of the maximum effect.

In a first analysis, the originally intact bridge is loaded by the dead weight and two side-by-side HS-20 trucks. The truck loads are incremented and a pushdown analysis is performed up to the complete failure of the structure. In a second set of analyses, eleven different damage scenarios are considered where in each scenario a different truss member is removed from the structure.

The analysis results show that the bridge behaves differently depending on the damage scenario, but overall the truss bridge shows a post linear capacity for most cases.

This document is divided into four sections. In the first section, the geometry of the bridge is described. In the second section, the results of the analysis are presented for the intact bridge. The third section shows the results of the damaged bridge scenarios. In the final section, the results of the damaged bridge scenarios are compared to the result of the pushdown of the originally intact bridge.
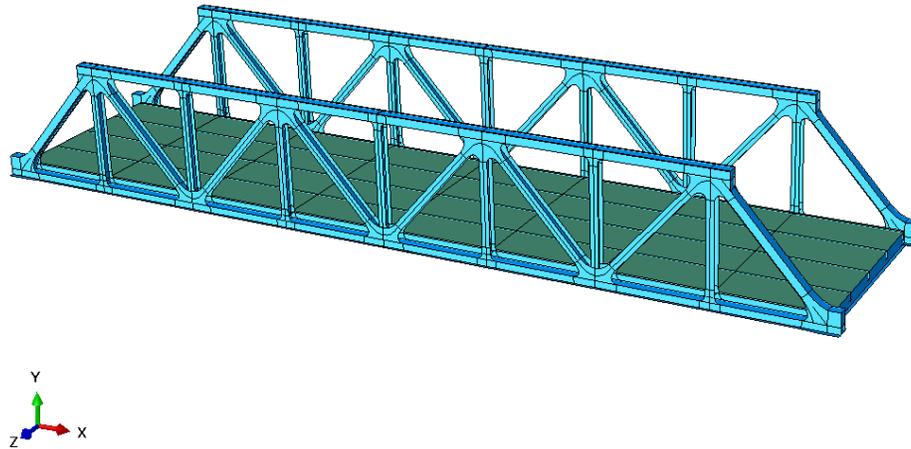
Figure 1- Original ABAQUS Model of Sample Truss Bridge

1. Bridge Description

A 3-D finite element model is used to analyze the behavior of the superstructure of the example truss bridge. The model is an adaptation of the Åby bridge truss in Sweden converted for vehicular type traffic. The bridge is 110 ft long and 24 ft wide. The geometric dimensions are shown in Figure 2. The truss's vertical members are spaced at 14 ft. Also, the main transverse beams are at 14'f from each other and the end beams are at 13 ft from the support. The truss is 14.5 ft high. The longitudinal deck stringers are spaced at 6.0 center to center. The concrete deck thickness is 6.5 in.

The applied live load is assumed to consist of two AASHTO HS20 trucks with typical axle load of 8, 32, and 32 kip respectively. The two trucks are assumed to be side –by-side and the most eccentric wheel load set is applied at 2.0 ft from the edge of the truss. The second wheel load is spaced 6.0 ft from the other wheel. The second truck load is spaced at 4.0 ft from the first truck. The transverse load location is shown in the bottom right corner of Figure 2.
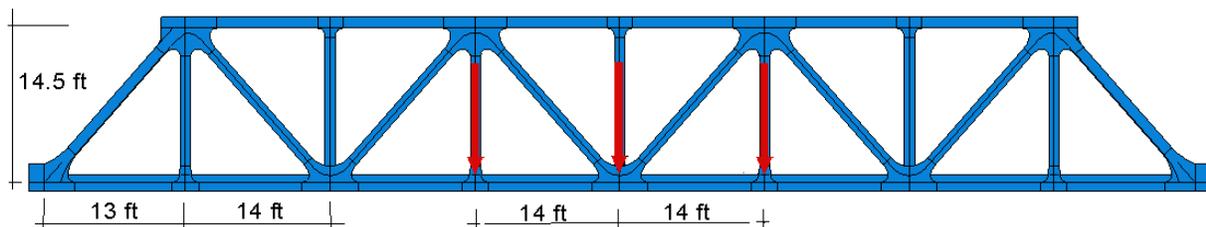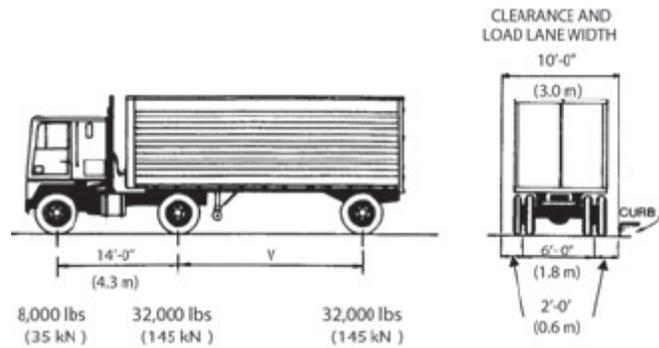

a) - Bridge Profile
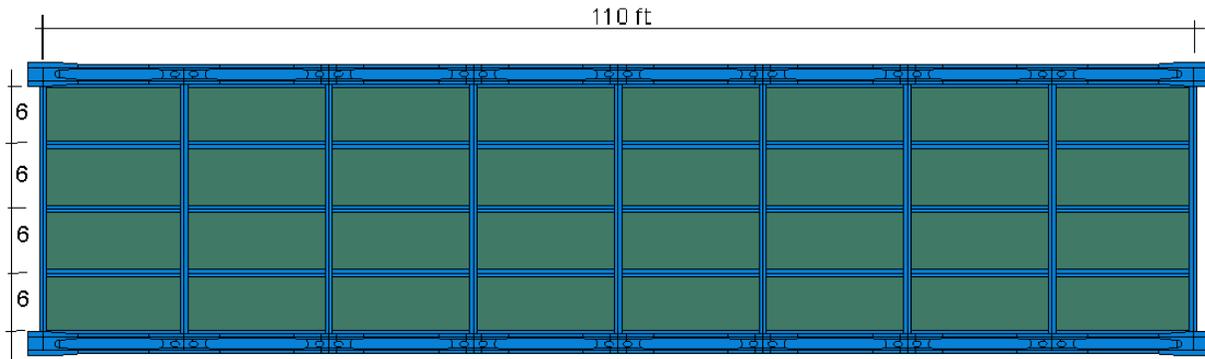
b) – Bridge Bottom View
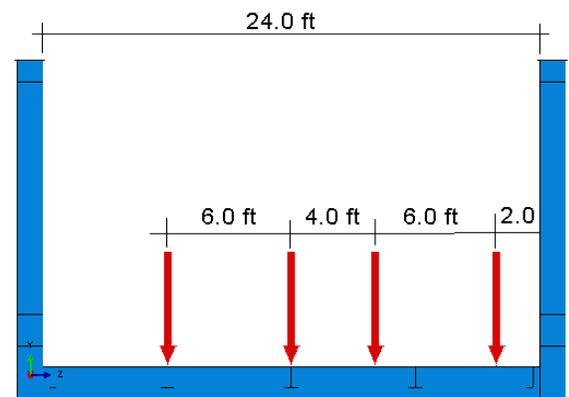
c) – Bridge Cross Section   d) HS-20 Live Load Configuration

e) Typical Aby Bridge Connection Detail

Figure 2- Bridge Geometry and Load Location Points

The steel truss members and the deck's cross beams and longitudinal beams are formed by assemblies of steel plates. The widths and thicknesses of the steel plates are listed in Table 1 for each member.

Table 1 – Steel Plates Dimensions

Truss Component	Cross section type	Width [in]	Thickness [in]
Vertical Flange (x2)	I-beam	12.0	0.80
Vertical Web		15.0	0.40
Diagonal Flange (x2)	I-beam	12.0	0.80
Diagonal Web		15.0	0.40
Compression Chord Flange (x 2)	box-beam	12.0	0.64
Compression Chord Web (x 2)		12.6	0.64
Tension Chord Flange Top	U shape	12.0	0.78
Tension Chord Flange Bottom		10.0	0.78
Tension Chord Web (x 2)		10.0	0.78
Deck			
Cross Beam Flange (x 2)	I-beam	8.3	0.43
Cross Beam Web		18.0	0.35
Longitudinal Stringer Flange (x 2)	I-beam	7.9	0.43
Longitudinal Stringer Web		12	0.35

The concrete deck strength is assumed to be 4.0 ksi. The stress strain curve for the steel is depicted in the graph of Figure 3. The Young's modulus is 29,000 ksi and the Poisson's coefficient is 0.3.
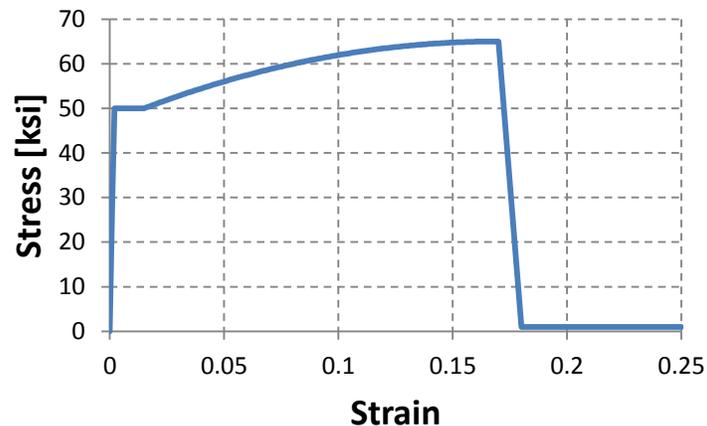

Figure 3 – Stress-Strain Curve for Grade 50 Steel

As shown in Figure 3, in order to account for local failures that may not necessarily affect the global response of the structure after local fracture, the steel's stress-strain curve is assumed to drop to a very small value of stress while keeping its ability to sustain additional strain after rupture. This will provide ABAQUS with the ability to represent the spread of plasticity after a local failure without encountering numerical instabilities.

While the proposed model accounts for the nonlinear behavior of the steel truss and girders, it is assumed that the bridge's concrete slab remains in the linear elastic range throughout the loading process. This assumption is made to catch the nonlinear behavior of the trusses alone. The concrete's Young modulus is assumed to be equal to 3,600 ksi and the concrete's Poisson ratio is equal to 0.2. These values correspond to a concrete strength f'$_c$=4.0 ksi.

The Von Mises model is suitable for most common steel static nonlinear analysis problems such as evaluating the ultimate load capacity of structural systems. The Von Mises plasticity model for structural steel is based on the Von Mises stress, σ_{VM}, which is obtained from the stress tensor at every point of the body as shown in Eq. 1:

$$\sigma_{VM} = \sqrt{\frac{1}{2}\left[(\sigma_X - \sigma_Y)^2 + (\sigma_Y - \sigma_Z)^2 + (\sigma_X - \sigma_Z)^2 + 6(\tau_{XY}^2 + \tau_{XZ}^2 + \tau_{YZ}^2)\right]}$$

(1)

Where σ_X is the normal stress component in the X direction, σ_Y is the normal stress component in the Y direction, σ_Z is the normal stress component in the Z direction, τ_{XY} is the shear stress component in the Y direction applied on the plane normal to the X axis, τ_{XZ} is the shear stress component in the Z direction applied on the plane normal to the X axis, and τ_{YZ} is the shear stress component in the Z direction applied on the plane normal to the Y axis.

To perform the nonlinear static analysis, ABAQUS applies increments to either the load applied on the structure (load control) or the displacement (displacement control). At every step, the program compares σ_{VM} of every element to check where it falls on the stress-strain curve shown in Figure 3. The program then adjusts the material's stiffness and checks whether the value of σ_{VM} is lower than the material's capacity. The program then traces the load-displacement curve for the control node selected by the user. In the analyses performed in this document, the control node is selected to be the node of the truss at which the maximum vertical displacement is measured.

2. Analysis of the Intact Truss Bridge

The push down analysis is performed on the originally intact bridge labeled as Model 1. The bridge is assumed to fail either when the structure reaches collapse or the maximum displacement reaches a value of span length/50 which is a very high value used to simply stop the analysis when a very large level of plasticity takes place. In this example, the bridge length is 110 ft and the displacement limit is 2.2 ft or 26.4 inches.

The program first applies the dead load to the structure that consists of the weight of steel trusses, the girders and the weight of the concrete deck; then it applies the live load due to the two side-by-side HS20 trucks. The live load is then incremented until the failure of the structure. According to Ghosn and Moses (1998) the acceptable level of redundancy depends on the type of failure considered. For an intact bridge, two redundancy measures can be evaluated: The first is related to the ultimate capacity of the system and the second is related to the functionality limit states. For the ultimate limit state, the load factor LF_u gives the number HS-20 trucks required to cause collapse and redundancy is measured as the ratio between LF_u and LF_1, where LF_1 is defined as the minimum value that causes the failure of the first member. LF_1 is equal to $(R - D)/LL$, where R is the member capacity of the most critical component, D is the dead load effect and LL is the live load effect of two HS20 trucks. The load factor corresponding to LF_1 for this bridge is equal to 7.00 indicating that the first member fails when each of the two HS trucks are loaded each up to a total weight equal to 72x7.00=504 kip or a total live load of 1008 kip. The value for LF_1 is calculated using the commercial software SAP2000 assuming linear elastic behavior of the truss as is done using traditional structural design and evaluation methods. For the functionality limit state, the redundancy is measured as the ratio between the LF_f which gives the number of HS-20 trucks needed to cause a maximum vertical displacement equal to span length/100 and LF_1 as defined earlier.

The result of the nonlinear analysis is depicted in the load deformation curve of Figure 4. The curve provides the maximum live load normalized to the weight of two HS20 side-by-side trucks (LF) versus the maximum vertical displacement. The behavior of the structure is globally linear until the point where the truss's compression chord starts to buckle although some portions of the structure have already started to yield, the yielding is still in its initial phase and thus the curve is almost linear until the onset of buckling. The buckling makes the structure deform without any significant additional increment in the applied load as the maximum displacement varies between 2 and 6 inch. After the deflection reaches 6 inch, the structure begins to unload but continues to deflect until collapse. The maximum load effect LF_u obtained for this configuration is equal to 9.45 times the effect of the two HS20 side-by-side trucks. The value of LF_1 is about 7.00 times the effect of the HS-20 live load. Hence, the redundancy ratio for the ultimate limit state is equal to 9.45/7.00 = 1.35 which is greater than the reference value of 1.30 as recommended in the NCHRP report 406 for the ultimate limit state.

When the maximum displacement reaches L/100 or 13.2 inches the maximum live load factor is about 9.0 and the redundancy ratio is equal to 9.00/7.00 = 1.28, which is greater than the reference value of 1.10 recommended in NCHRP 406 for the functionality limit.

Figures 5 and 6 show the effect of the buckling on the structure and the area of the bridge where the material yields. It is noted that the bridge is able to redistribute the load to regions far away from the area

where the buckling is concentrated. This is possible when the connections between the two parallel trusses and the deck are sufficiently strong to allow for the redistribution of the load from the more heavily loaded truss to the other truss. .

According to the criteria defined in NCHRP report 406, the intact bridge structure analyzed in this example can be considered redundant.
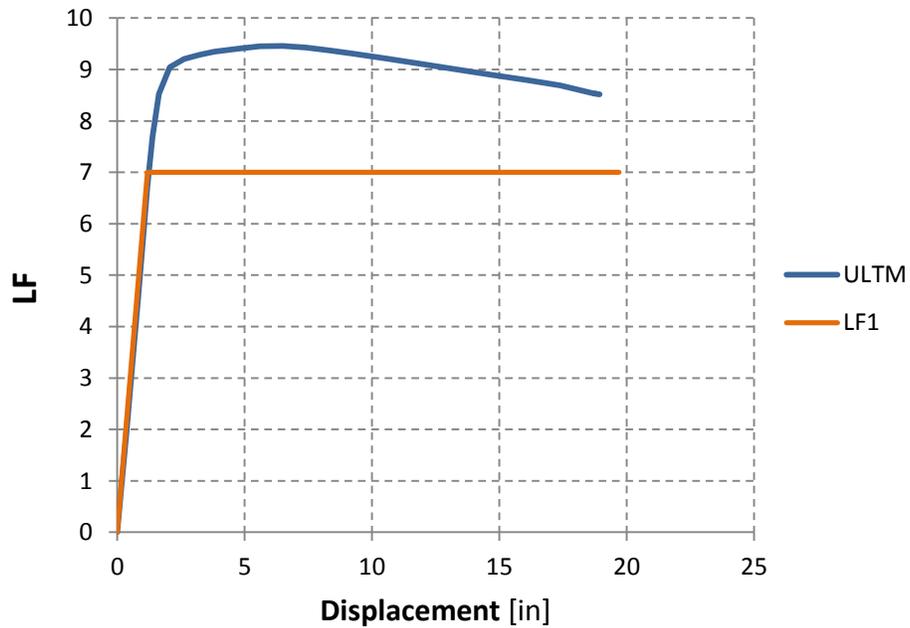
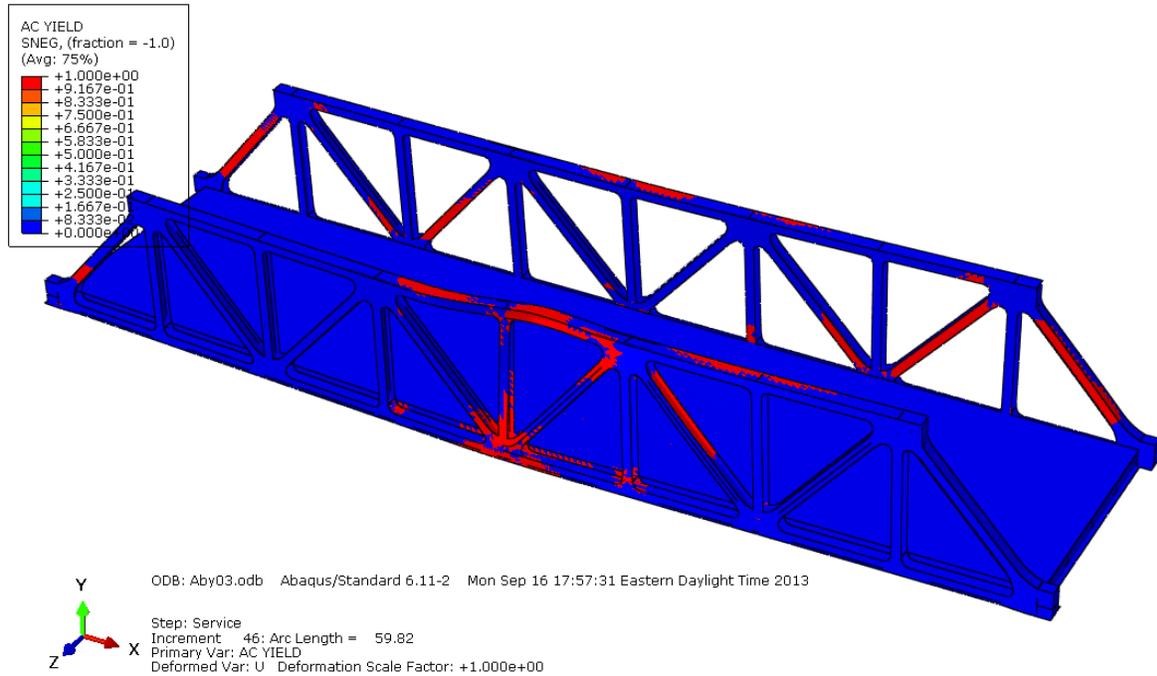Figure 4 – Model 1 – Intact Structure Displacement vs Normalized Vertical Load

Figure 5 – Model 1 – Intact Structure Plastic Regions
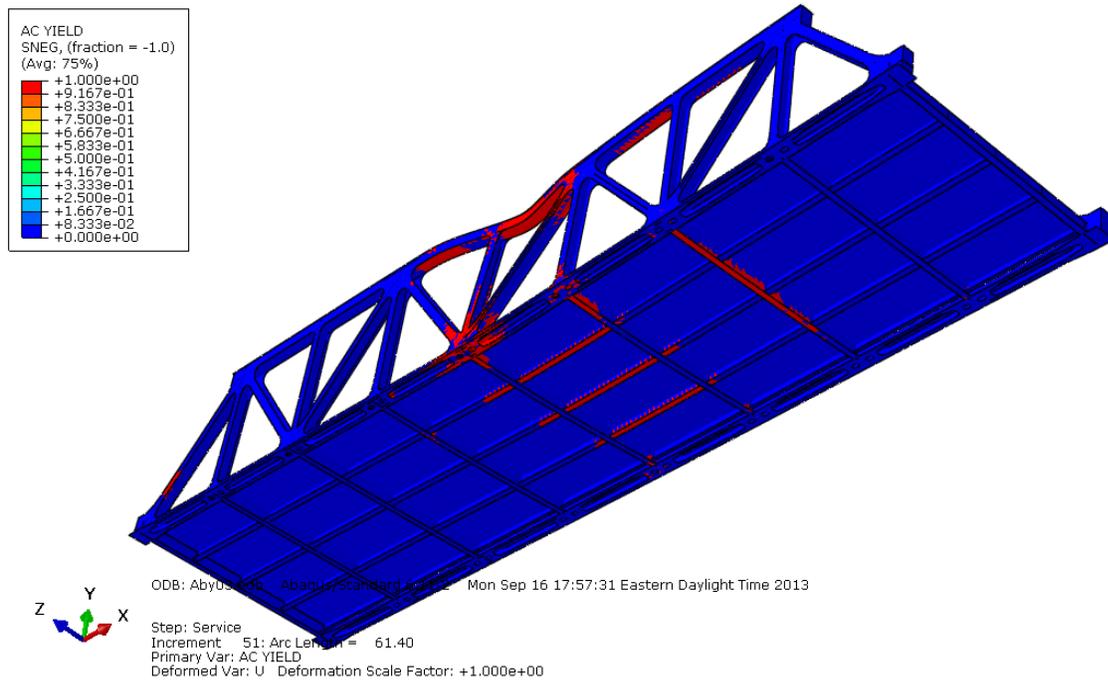
Figure 6- Model 1 – Bottom View of Intact Structure Showing the Buckling Failure

3. Analysis of Damaged Bridge

In this section, the bridge is analyzed assuming different damage scenarios. The damages are applied to the truss that is close to the vehicle loads, which is the truss that failed during the analysis of the originally intact bridge. Damages in all four types of truss members are considered: 1) the compression chords on the top of the truss; 2) the tension chords in the bottom part of the truss; 3) vertical members and; 4) to the diagonals. In all cases, the damage is simulated by removing an entire member and the pushdown analysis is performed on the modified models. This would simulate the fracture of tension members due fatigue or the failure of any member due to an impact. It is noted that the analysis is performed to estimate the post-damage capacity of the system under static loading. The analysis does not consider the dynamic behavior during impact or during the release of the fracture energy. The purpose of the analysis is to verify that a damaged bridges will still be able to carry a sufficient level of traffic until the damage is noticed, the proper authorities alerted and appropriate decisions on repair or closure are taken.
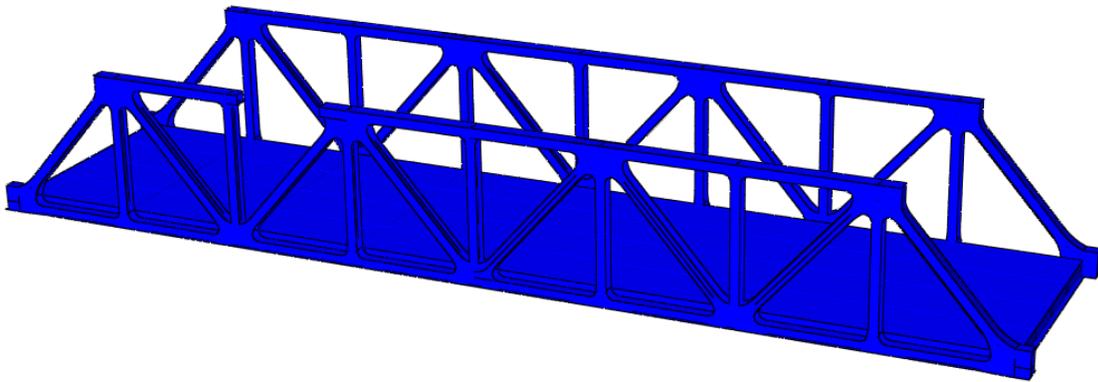
Model 2: Damage of Compression Chords

When the damage is applied at the compression chord, three cases have been considered, one for each compression member from the support through the middle span as shown in Figure 7. In the first case, the compression member is removed in the part of the truss closest to the support (CM01). Subsequently, in the two other cases CM02 and CM03, a different compressive chord member is removed one at a time.
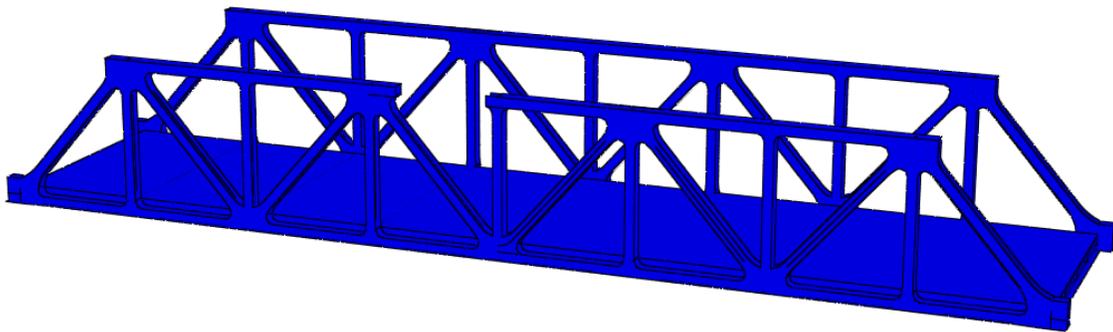
CM01
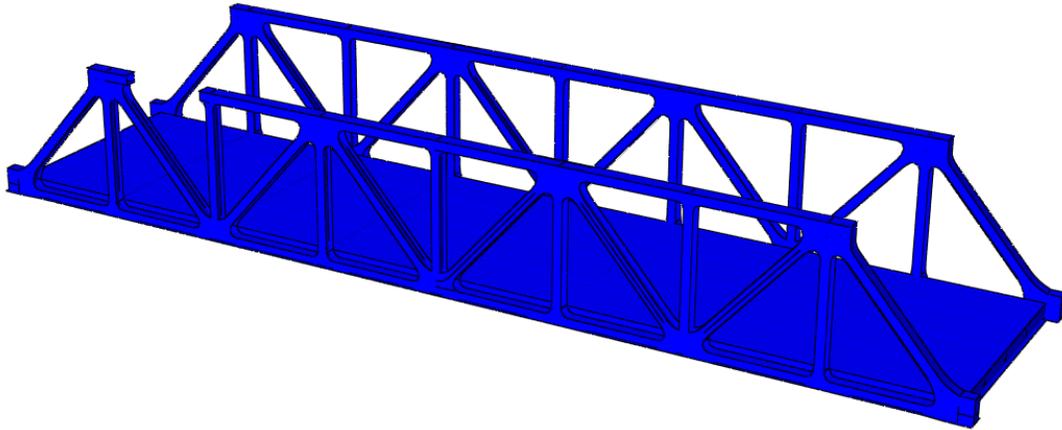
CM02

CM03

Figure 7- Model 2 – Damaged Structure of Compression Chords (CM01, CM02 and CM03)

The results of the analysis of the cases of compression chord member damage show that when one member of the compression chord is removed, the overall stiffness of the truss is drastically compromised and the stiffness is reduced compared to the stiffness of the intact bridge. However, despite the lower stiffness, this particular bridge configuration is able to carry significant live load after damage. In fact, the bridge is able to reach a capacity of two times the effect of two side-by-side HS20 trucks before reaching the functionality limit state as shown in Figure 8. This is possible because the connections between the truss members of this bridge can carry a substantial level of moment and do not behave as pins. A typical connection for this bridge is shown in Figure 2. Because of the connection type, the bridge is able to redistribute the load to other elements of the structure through moments. Figure 9 is provided to help visualizes this aspect of the load redistribution process which shows how plasticity spreads to the members close to and far away from the damaged portion of the chord. The span/100 displacement of the damaged bridge is reached at a load increment of 2.80 trucks for cases CM01 and CM02 and 1.50 for CM03. The ratio of these loads compared to LF_1 are 2.80/7.00 = 0.40 for CM01 and CM02 and 1.50/7.00 = 0.21 for case CM03.
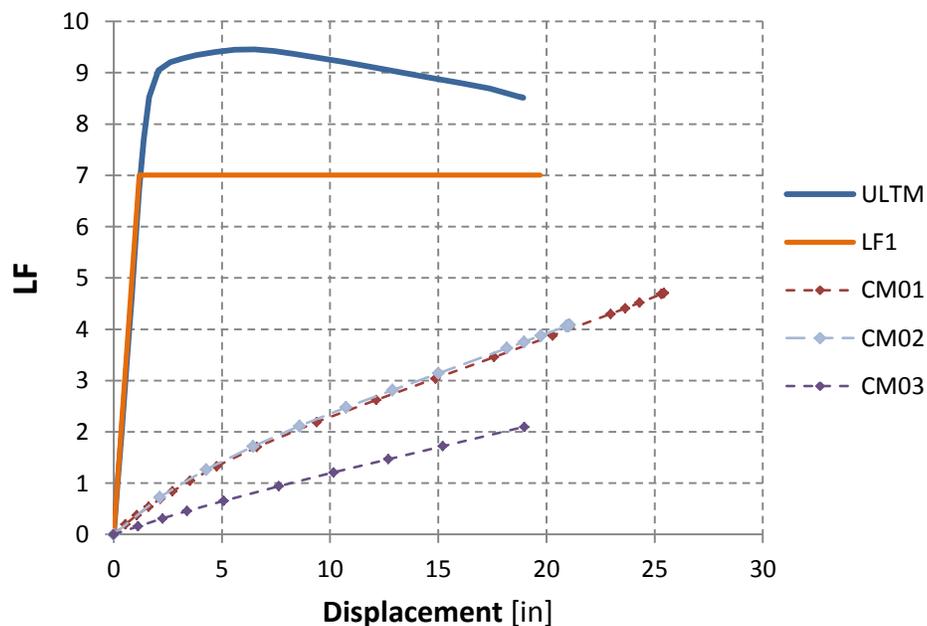

Figure 8- Model 2 – Damaged Structure Displacement vs Normalized Vertical Load (CM01-CM03)
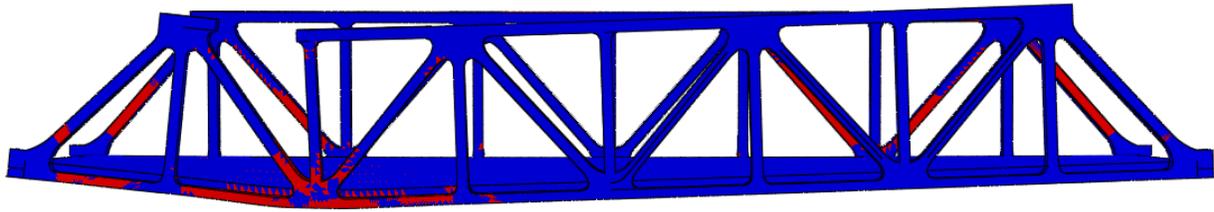
Figure 9- Model 2 – Top Chord Damaged Structure Moment Effect of the Connection

At the damaged limit state NCHRP 406 requires that the redundancy of the damaged bridge be equal or greater than 0.50. The normalized vertical load factor LF obtained for the damaged top chord scenarios are 2.09 for case CM03, 3.57 for CM02 and 4.68 for CM01 as shown by the load deflection curves of Figure 8. The damage redundancy ratio varies from 2.09/7.00 = 0.30 through 4.68/7.00 = 0.66. According to NCHRP 406, the bridge is non-redundant if it is subjected to a damage of the type of CM03 which removes the middle top compression chord, while a bridge subjected to the damages of types CM01 and CM02 can be considered redundant.

For damage scenarios related o the compression chord the final failure of the bridge happens by rupture of the tension members in the vicinity of the damage as shown in Figure 10.
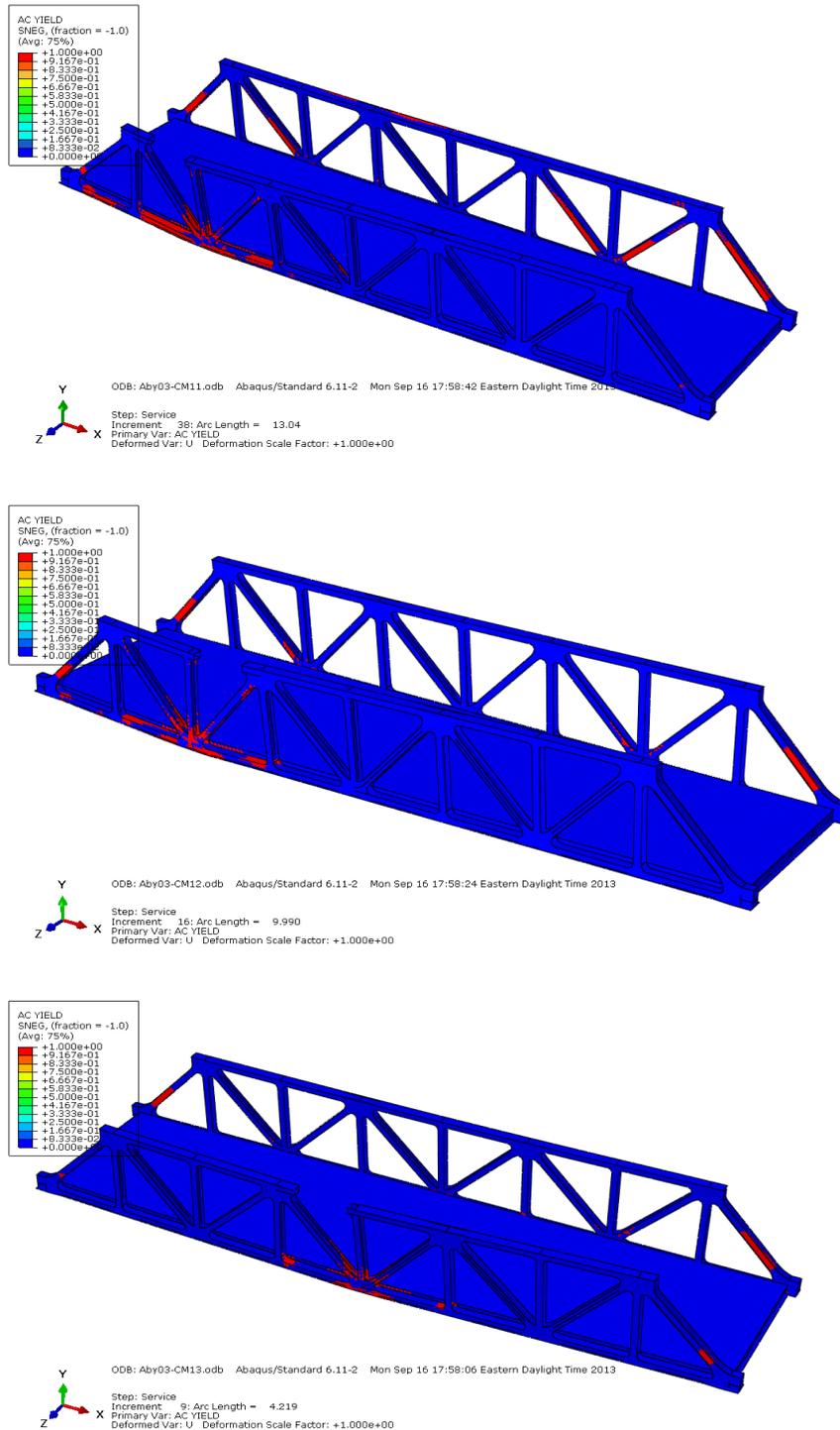
Figure 10- Model 2 – Top Chord Damaged Structure Plastic Failure

Model 3: Damage of the Tension Chord

In this damaged configuration, the tension chord close to the middle point of the bridge was removed (TM01). The damaged bridge configuration is shown in Figure 11. This type of damage can simulate the scenario of the member loss due to fatigue fracture.
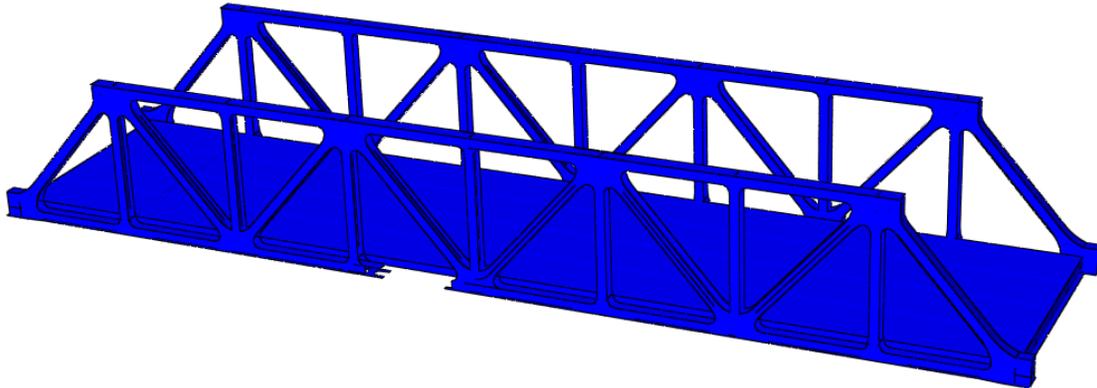

Figure 11- Model 3 – Damaged Structure of Tension Chord (TM01)

The results of the pushdown analysis shows that the loss of a tension member in the lower chord does not affect substantially the performance of the structure, as shown in the load versus deformation curve in Figure 12. This happens, because the deck can provide sufficient capacity to carry the load that was originally in the missing member. Both the stiffness and ultimate capacity of the bridge system are comparable to those of the intact bridge configuration.
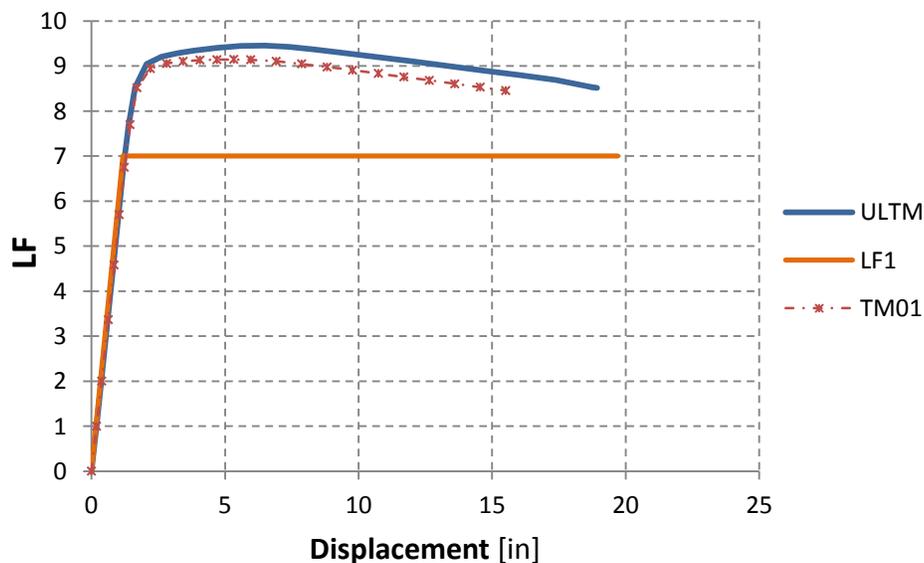

Figure 12- Model 3 – Damaged Structure Displacement vs Normalized Vertical Load (TM01)

The redundancy ratio for the load that causes a deflection equal to span length/100 is equal to 8.60/7.00 = 1.23, while the redundancy ratio for the damage limit state is equal to 9.14/7.00 = 1.31 which is much greater than 0.50 which was proposed in NCHRP report 406. This effect as noticed earlier is the consequence of the fact that the damage of the tension does not significantly affect the behavior of the truss because the deck can replace the original contribution of the missing member.

The failure of the truss as was observed in the intact configuration happens due to the buckling of the compression chord as shown in Figure 13.
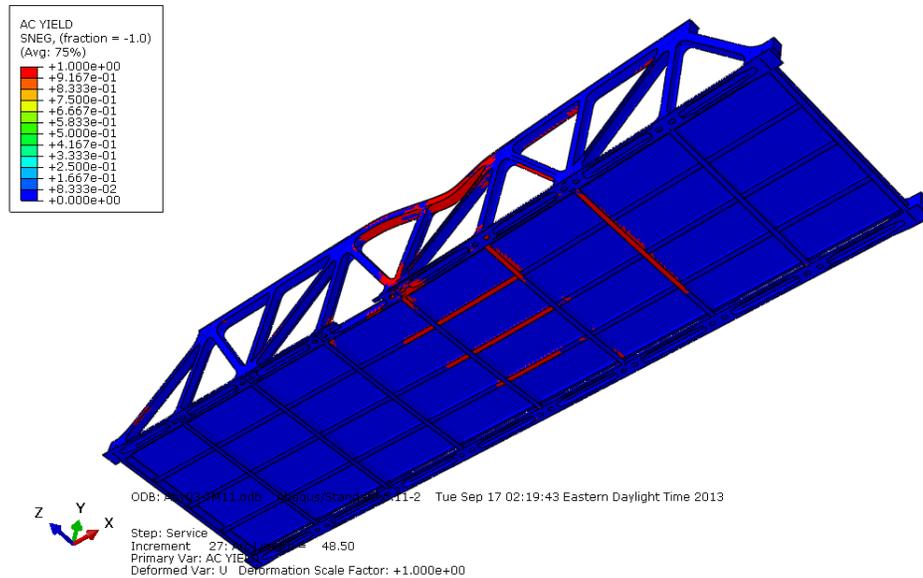

Figure 13- Model 3 – Bottom View of Tension Member Damaged Structure Showing Buckling Failure

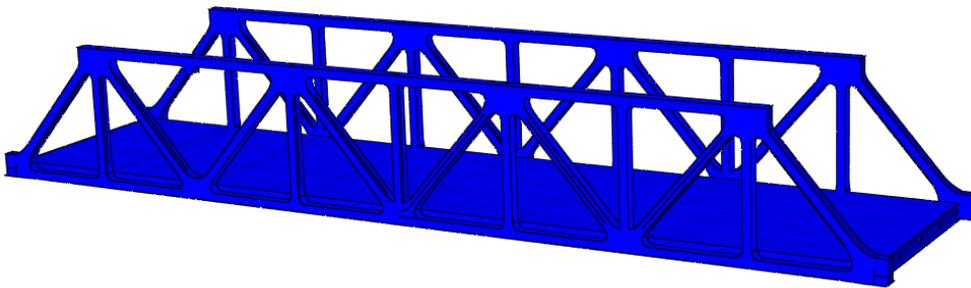*Model 4: Damage of Vertical Members*

For this damage type, four different scenarios have been considered. One vertical member at a time is removed for each of the four cases as shown in Figure 14. In the first case, the diagonal member is removed in the part of the truss closest to the support (VM01). In the three following cases VM02, VM03 and VM04 are removed one at a time. The loading is still kept at the middle of the bridge because this happens to gives the lowest capacity for the system.

The results of the analysis for this type of damage shows that when one vertical member is removed, the global stiffness of the truss is not compromised compared to that of the intact bridge while the maximum capacity is slightly reduced as shown in the load displacement curves of Figure 15. The bridge finally fails due to the buckling of the compression chord; this is depicted in the failure mode of Figure 16.
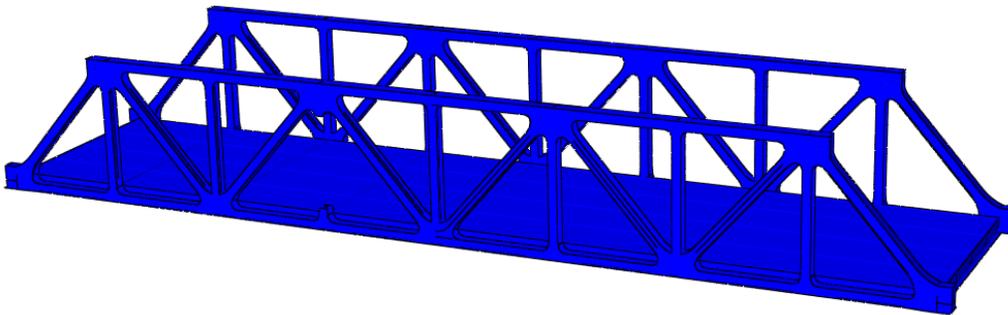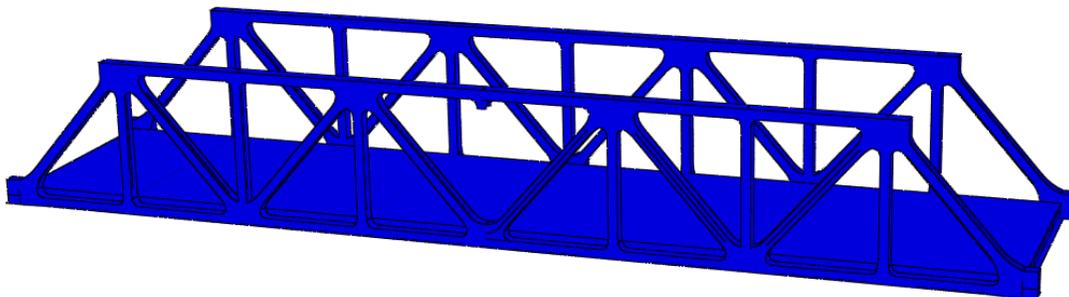
VM01
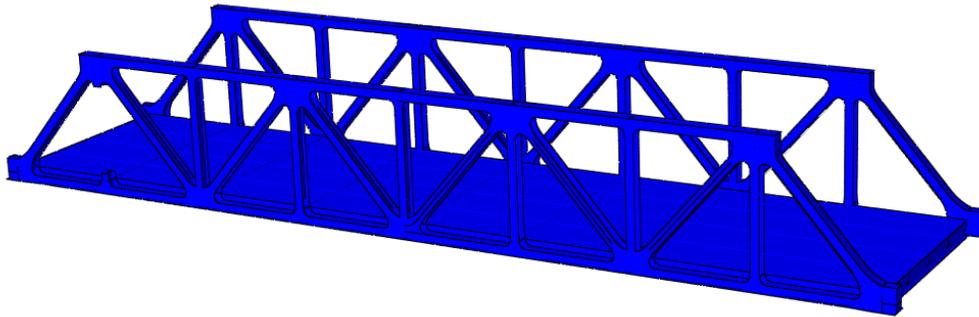
VM02

VM03

VM04

Figure 14- Model 4 – Damaged Structure of Vertical Members (VM01, VM02, VM03 and VM04)

When a vertical member near the support is damaged (VM01 and VM02), the behavior of the truss is practically the same as that of the intact structure. This means that the two vertical elements for the specific load condition do not contribute significantly to the distribution of the load. The redundancy ratios at span length/100 and for the maximum load for the damage limit states are equal to the redundancy values of the intact bridge at the functionality and ultimate limit states with LF ratios are equal to 1.28 and 1.35 respectively.

When the damaged element is closer to the load location (middle of the span), the effect of the damage becomes more significant. The maximum capacity and the load at span/100 are reduced compared to the originally intact bridge. In fact, the VM03 and VM04 curves in Figure 15 show a reduction in the capacity compared to that of the originally intact system, while cases VM01 and VM02 show similar results as those of the originally intact system

The redundancy at span length/100 ranges between 7.97/7.00 = 1.14 for the damage condition VM04 and 9.00/7.00 = 1.29 for cases VM01 and VM02. The damage bridge redundancy ratio varies from 8.90/7.00 = 1.27 through 9.45/7.00 = 1.35. According to the NCHRP 406 the structure for all vertical member cases can be considered redundant.
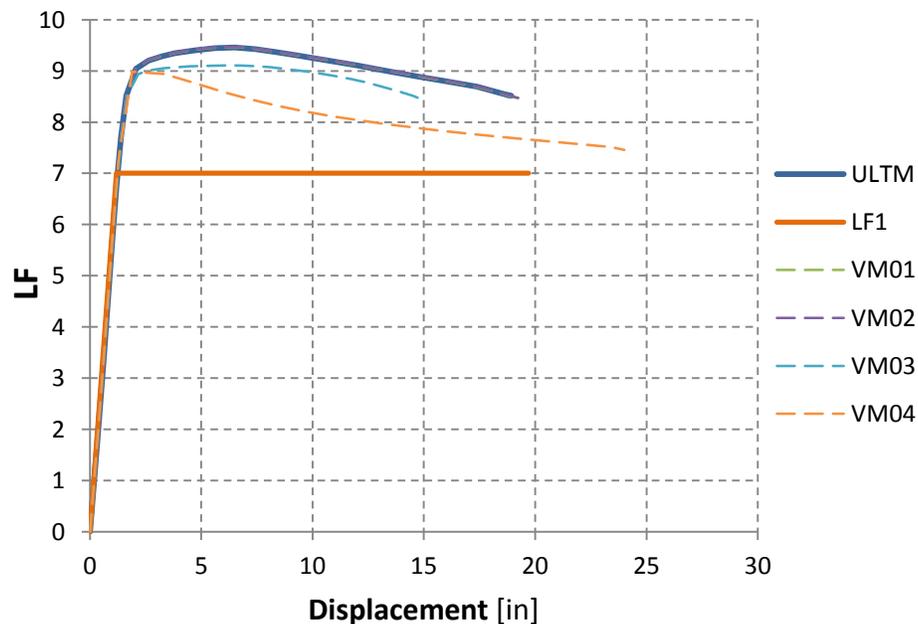

Figure 15- Model 4 – Damaged Structure Displacement vs Normalized Vertical Load (VM01 – VM04)
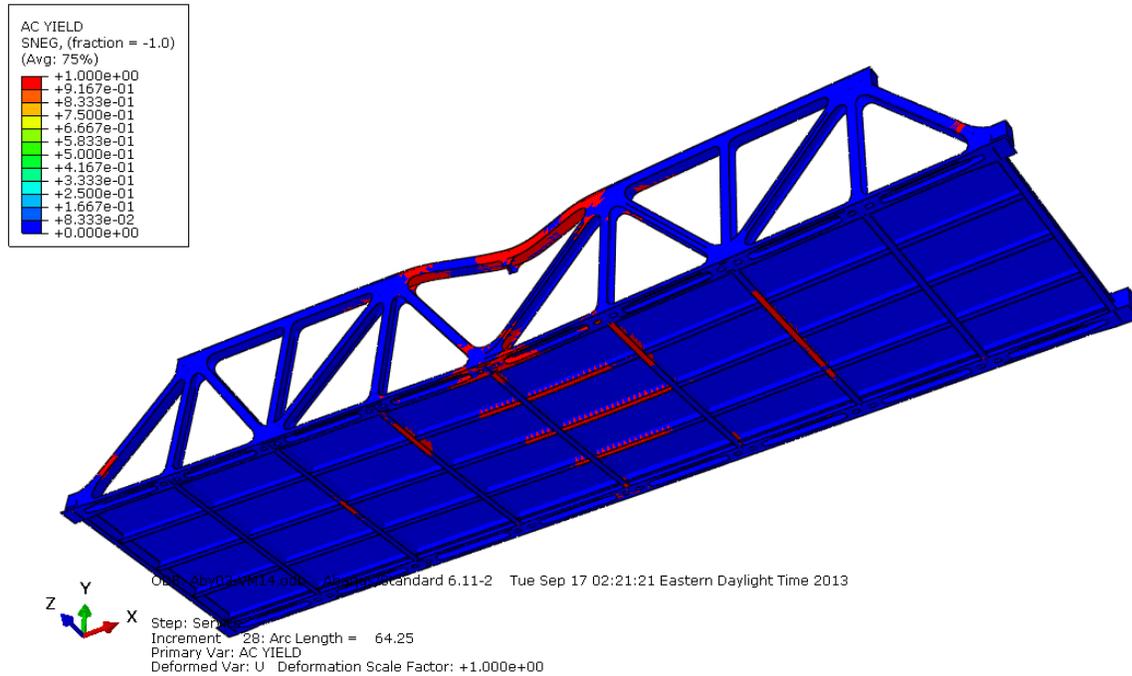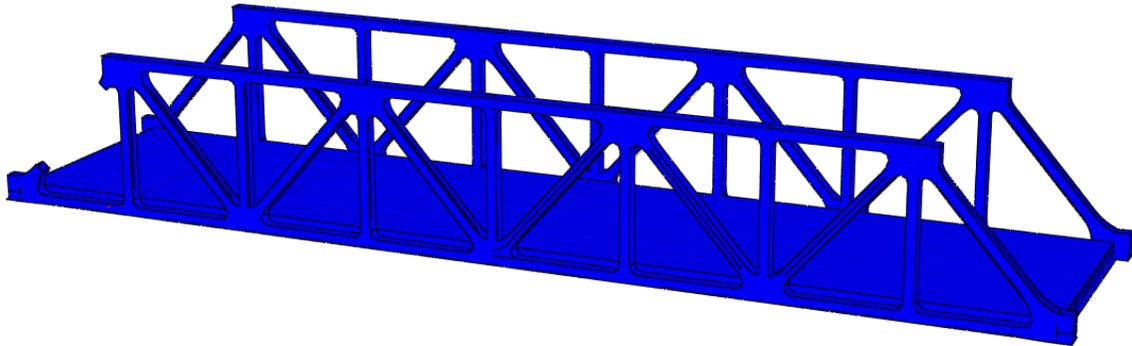
Figure 16- Model 4 – Bottom View of Damaged Structure Showing Buckling Failure

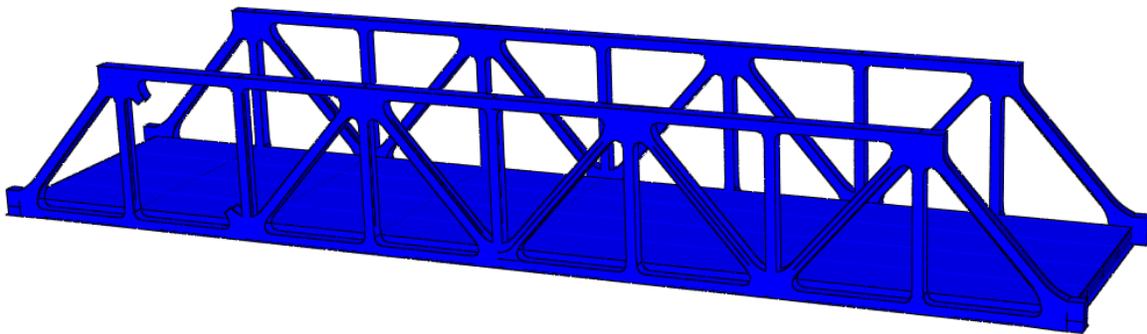*Model 5: Damage of Diagonal Members*

For this damage scenario, three cases have been considered. One diagonal member at time is removed for each of the four cases as shown in Figure 17. In the first case the diagonal member of the truss closest to the support is removed (DM01). Subsequently, in the two cases DM02, and DM03, a new diagonal member is removed after restoring the previously eliminated member.

The results of the analysis of this type of damage show that when the diagonal is removed at the support, the global stiffness is affected. In fact, for this case the truss effect is missing and the entire load is transferred through the connection at the bottom to the truss away from the load as shown in Figure 18. For the remaining cases, the stiffness is not compromised as much as in case DM01 although the stiffness is lower than that of the intact scenario. For these cases, the effect of redistribution of the load to a wider portion of the structure allows the bridge to reach an ultimate capacity greater than that of the intact bridge. This is possible because the failure mechanism is not localized in one portion of the truss and plasticity distributes throughout the system and is not concentrated in the region close to the load as in the intact configuration that produces buckling of the compression chord before the load redistributes to regions away from the load. The damage and the failure for cases DM02 and DM03 are shown in Figure 19 while the results of the analysis represented by the load versus displacement curves are shown in Figure 20.
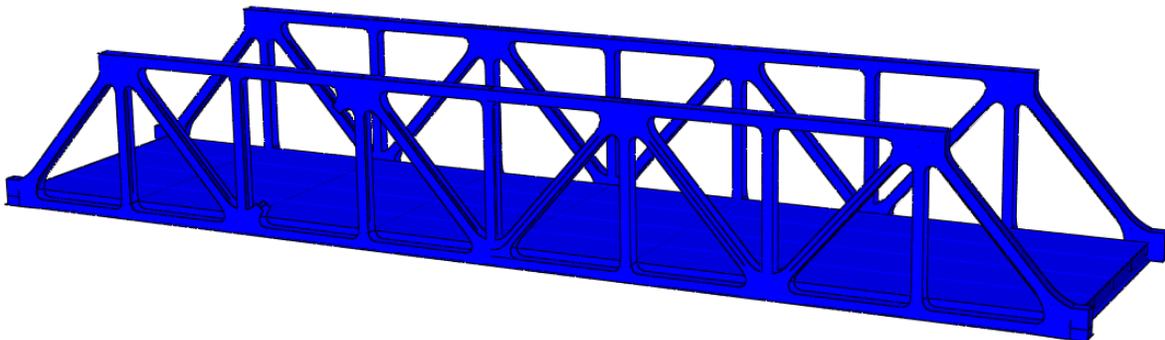
DM01

DM02

DM03

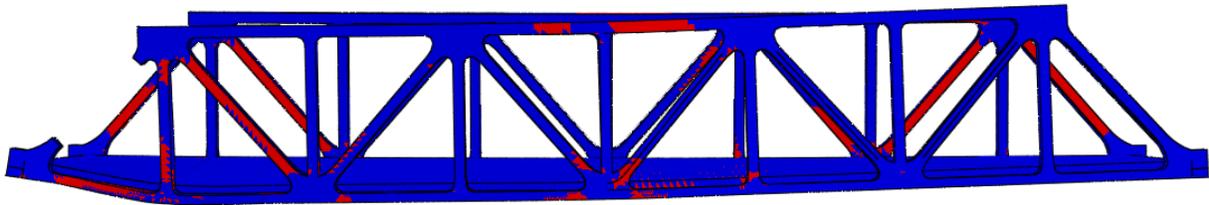Figure 17- Model 5 – Damaged Structure of Diagonal Members (DM01, DM02, DM03 and DM04)

Figure 18- Model 5 – Damaged Structure Moment Effect of the Connection (DM01)
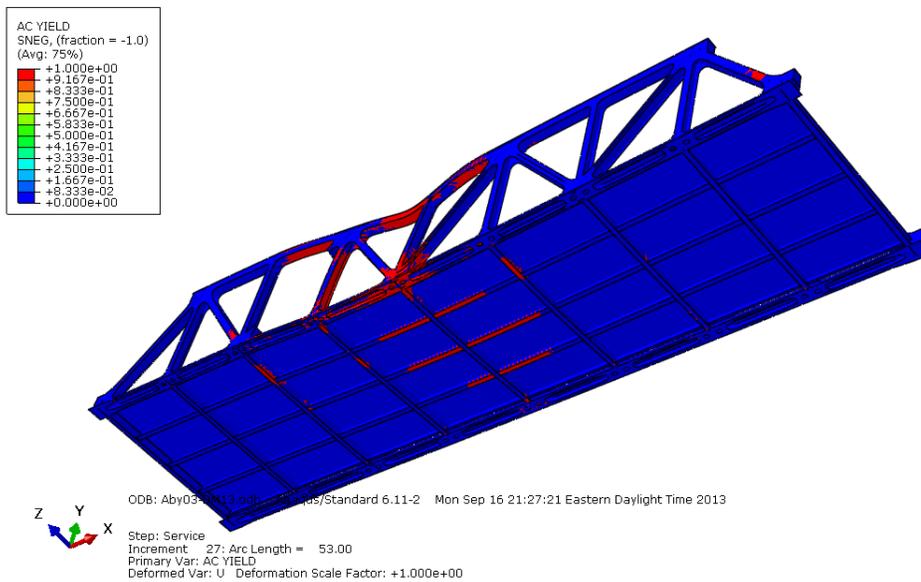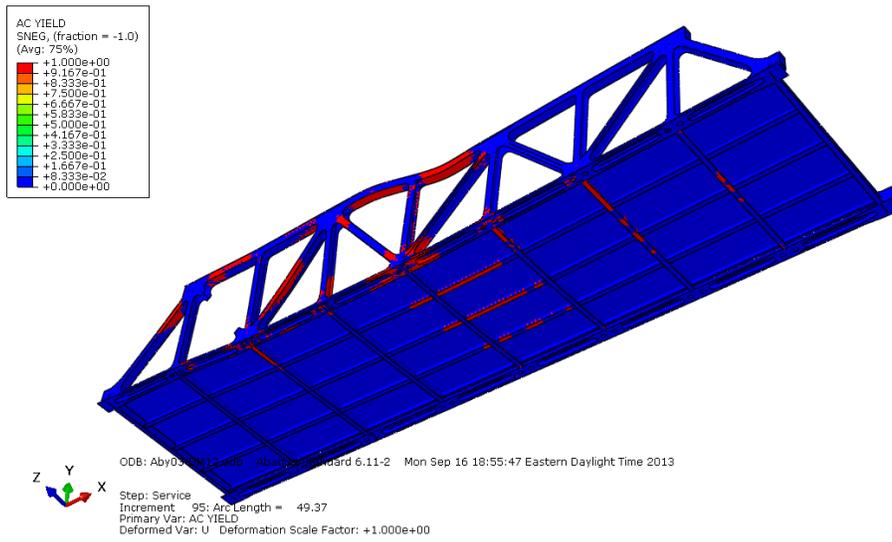

Figure 19- Model 5 – Bottom View of Diagonal Damaged Structure Plastic - Buckling Failure
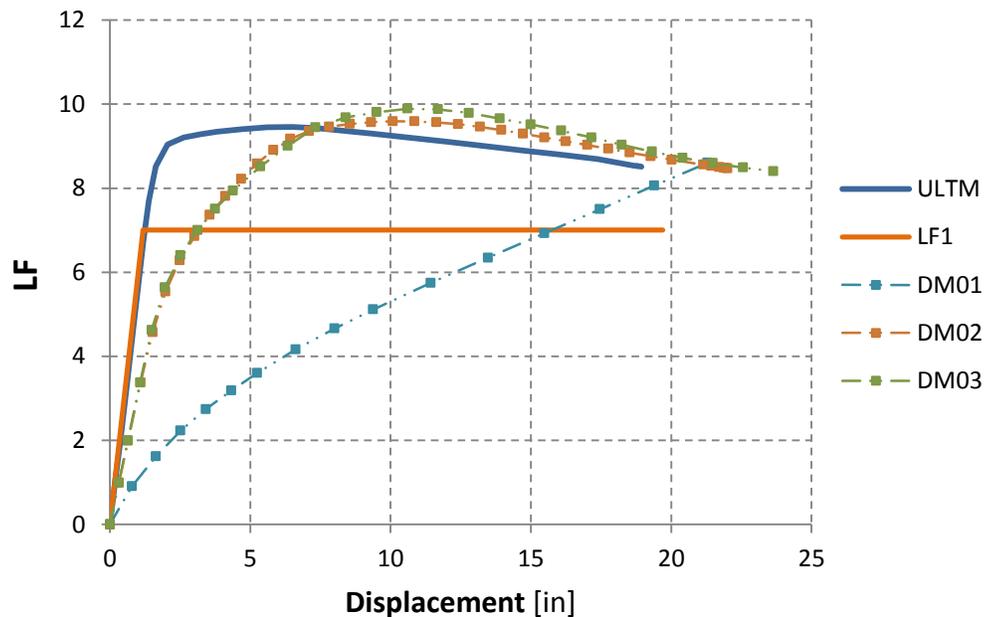
Figure 20- Model 5 – Damaged Structure Displacement vs Normalized Vertical Load (DM01 – DM03)

The redundancy ratio for case DM01 is equal to 6.34/7.00 = 0.91 for the span length/100. The redundancy ratio for the damaged limit state is equal to 8.60/7.00 = 1.23 that is much greater than the 0.50 criterion proposed in NCHRP 406.

At the span length/100, the redundancy ratios for damage scenarios cases DM02 and DM03 are equal to 9.46/7.00 = 1.35 and 9.78/7.00 = 1.40 respectively. The redundancy ratio for the damaged limit state is equal to 9.60/7.00 = 1.37 for case DM02 and 9.89/7.00 = 1.41 for case DM03. The bridge is considered to be redundant for these two damage scenarios according to the NCHRP 406 criteria.

4. Comparison of Results

The analysis of different daamage scenarios for the exmple truss analyzed in this Report showed that the redundancy ratio depends on the location where the damage develops. In fact, the ratio for different limit states varies considerably. Table 2 summarises the results of the different damage scenarios.

Table 2 shows that the laod at which the span length/100 dispalcment is reached varies from a minimum of 0.21 for case CM01 to a maximum of 1.40 for case DM03, while the maximum capacity ratio varies from a minimum of 0.30 for case CM01 to a maximum of 1.41 for case DM03.

It is noted that local failure of each member of a damaged truss depends on the load location. However, our analysis cncenrtated on the global behavior and the laod was applied in the position that prodcued the crictial damaged system capcity. If one wishes to study the local behavior of the truss members, then

different laoding positions should be used for the different damage scenarios. That would be needed to identify which members must be strengthened to improve the global behavior of the damaged systems.

The good perforamnce of his system is related to the types of connections used in this truss and the ability of the deck to redistrbute the load both of these factors can drastically affect the global behavior of the system.

Table 2 – Redundancy Ratios for Aby Bridge (shaded cell shows low redundancy level)

Analysis Case	LFu/LF1 Ultimate limit state of originally intact bridge	LFf/LF1 Functionality limit state of originally intact bridge	LFd/LF1 Redundancy ratio for damaged bridge scenarios	LF100/LF1 for damaged bridge scenarios
ULTM	1.35	1.29	-	
TM01	-		1.31	1.23
CM01	-		0.30	0.21
CM02	-		0.57	0.40
CM03	-		0.67	0.40
VM01	-		1.35	1.29
VM02	-		1.35	1.29
VM03	-		1.30	1.25
VM04	-		1.27	1.14
DM01	-		1.23	0.91
DM02	-		1.37	1.35
DM03	-		1.41	1.40

CONCLUSIONS

The analysis of the system redundancy of truss bridges is a complex process that requires an exhaustive investigation in order to characterize all possible damage scenarios and the response of the bridge to these cases under different loading conditions.

The analysis of the sample brige presented in this report shows that the redundancy is strongly dependent of the location of the damage. For example, the originally intact bridge provides sufficient levels redundancy for both the functionality and ultimate limit states according to the criteria proposed in NCHRP report 406.

For the damaged scenarios, the values of the redundancy ratios vary in function of the location of the damage. As an example, the damaged case redundancy ratio varies from a minimum of 0.30 for case CM03 which assumes that the compression top chord member near the middle of the bridge is damaged through a maximum of 1.41 for case DM03 which is assuems that a diagonal member near the middle of the span is damaged.

The type of connections used between the truss members and the connections bewteen the trusses and the deck affect drastically the global behavior of the truss system.